

**FOUNDATION INVESTIGATION AND DESIGN REPORT
GWP 392-98-00
PROPOSED EMBANKMENT WIDENING
HIGHWAY 17 FROM STATION 30+750 TO 31+100
TOWNSHIP OF HEAD**

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1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by JMC Transportation Group to conduct foundation investigations at nine locations for the proposed embankment widening for the passing lanes of Highway 17 and a culvert extension in the Townships of Head and Rolph. This report summarizes the factual information and provides recommendations for one of these sites.

Authorization to proceed with this work was received from JMC Transportation Group. This work was carried out as part of their Total Project Management Contract for the Ministry of Transportation of Ontario (MTO) under G.W.P. 392-98-00.

The project is located on Highway 17, from 6.2 km west of Renfrew County, Road 635, easterly 18.6 km. As part of this project, it is proposed to construct one new east bound passing lane from Station 28+250 to Station 30+850, Twp of Head, one new westbound passing lane from Station 29+120 Twp of Head to Station 12+250 Twp of Rolph and to extend the Colton Creek culvert. Within the alignment for the passing lanes, Detailed Foundation Investigation and Design are required at nine locations for embankment widening and for a culvert extension. The specified locations for detailed foundation design are as follows:

Township of Head	Station 28+475 to 28+700
Township of Head	Station 29+175 to 29+350
Township of Head	Station 30+025 to 30+150
Township of Head	Station 30+450 to 30+600
Township of Head	Station 30+750 to 31+100
Township of Rolph	Station 10+400 to 10+675
Township of Rolph	Station 11+250 to 11+550
Township of Rolph	Station 11+725 to 11+825

Township of Rolph Station 12+050 to 12+250

At the proposed passing lane locations, the through lanes and the passing lanes will be 3.75 m wide. Shoulder widths at the passing lanes will be 3.0 m on the side of the new passing lane and 2.3 m on the opposite side. The shoulder rounding on the side of the new passing lane will be 1.0 m.

This report addresses the field investigation, laboratory testing program and design for the proposed widening at the Township of Head, Station 30+750 to 31+100, which includes a culvert extension at Station 30+930, as defined by the Ministry as the Foundation Investigation Report.

2.0 SITE DESCRIPTION

Along this section of highway under investigation for the proposed widening, the existing highway embankment varies in height from approximately 1.8 m to 2.9 m within the proposed construction. The embankment will be widened on the north side.

The ditch along the north side of the highway within the limits of investigation generally is grass covered to the east of the culvert and to the west, cattails exist. The site slopes downward from east and west toward the culvert. The culvert is a 3.65 x 1.7 x 22.7 concrete rigid frame culvert. The north tree line at the culvert location is approximately 23 m from centreline of highway. The culvert diverts Colton Creek under the highway, flowing south to north. An abandoned foot bridge exists 12 m north of the culvert.

A picture of the site taken from Station 30+750 looking east is shown below.



3.0 INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between November 11 and December 9, 2004 utilizing a CME 750 and track mounted super 75 drill rig equipped for geotechnical drilling and operated by DST. Fifteen boreholes were put down to depths ranging between 2.7 to 10.2 m. In addition, two dynamic cone penetration tests (DCPT) were conducted in the vicinity of the culvert at Station 30+930.

Borehole locations and a stratigraphic profile and section are shown on the Borehole Location Plan, Drawing No. 1. Boreholes 32, 32A and 34 are located in the shoulder of the roadway while Boreholes 35 to 39 and 41 to 45 are located between the toes of the existing and proposed embankments. Boreholes 33 and 40 and DCPT 1 and 2 were put down at the culvert location to provide soil data for the proposed extension. All boreholes were advanced with hollow stem augers to auger refusal which varied from 2.7 to 10.2 m below existing grade.

Auger refusal occurred in the highway fill at a depth of 1.0 m. Numerous attempts were made in the immediate area but could not penetrate past 1.0 m. The borehole was moved west from Station 30+870 to Station 30+797 where penetration through the fill was achieved.

Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 50 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 300 mm is known as the standard penetration blow count (N) which provides an indication of the denseness or consistency of the soil. Representative soil samples are obtained from within the sampler. Borehole Logs are presented as Enclosures 1 to 17.

Ground surface elevations at the borehole locations were surveyed by K. Smart Associates Limited.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, supervised the drilling, sampling and in-situ testing, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included natural moisture contents, gradation analyses and organic content. Laboratory test results are presented on the Borehole Logs and Enclosures 18 to 20.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Published Engineering Geology

The Quaternary and bedrock of the area under investigation as reported by John F. Gartner and P.F. VanDine in the Northern Ontario Engineering Geology Terrain Study 103, Deep River Area (NTS31K/SW) District of Nipissing and County of Renfrew consists of a discontinuous veneer of ground moraine till over the bedrock. During deglaciation, a number of glaciofluvial deposits were formed (Chapman 1975). Precambrian rocks of the Grenville Structural Province underlie the Deep River map area. The oldest and most abundant rocks of the metamorphic complex are metasediments derived largely from siliceous sandstone and siltstones. These metasediments consist of a variety of gneisses and can be found throughout most of the map area.

The mapping associated with the above report indicates that the soil, landform, topography and drainage generally consist of sand and gravel outwash plain with moderate local relief and dry.

4.2 Stratigraphy Overview

The generalized stratigraphy of the site based on the borehole locations off the highway (Boreholes 35 to 38 and 42) consist of an organic layer overlying sand and silt. At Boreholes 43, 44 and 45, the organic layer consists of topsoil. A discontinuous silty clay seam was identified at Borehole 44 starting at a depth of 0.75 m and extending to a depth of 1.5 m. Auger refusal occurred in all boreholes. The refusal material was not confirmed by diamond drilling techniques, therefore could be boulders or bedrock.

The highway fill, as identified by Boreholes 32, 32A, 33 and 34, consists of granular fill varying between 2.3 and 3.8 m deep. Below the granular fill to auger refusals of 6.8 m to 10.2 m a sand and/or silt exists.

The stratigraphy at the culvert location was identified by Boreholes 33 and 40 and DCPT 1 and 2. Borehole 33 is located on the highway and consists of highway granular fill with trace organics to a depth of 2.3 m which is underlain with sands and silts. Borehole 40 is located approximately 4.5 m beyond the end of the existing culvert and consists of fills comprised of topsoil and sand with organics to a depth of 2.1 m. The fill is underlain by sands and silts.

4.3 Embankment Fill Within the Shoulder

Fill within the embankment shoulders consists of sand and gravel to silty sand as indicated at Boreholes 32, 32A, 33, and 34. The granular fill comprises the base and subbase materials and varies in thickness from 2.3 to 3.8 m. The base materials vary in thickness from 0.3 to 0.4 m. The bottom of the fill varies between elevations 169.3 m and 170.4 m.

The Standard Penetration Test (SPT) results generally indicate very loose to compact state of denseness (N values vary from 1 blows/0.3 m to 11 blows/0.3 m).

The granular fill generally consists of sand with varying amounts of silt and gravel. At Borehole 33, near the existing culvert, the sand fill contained trace amounts of organics. Rockfill was noted in Borehole 34 at a depth of 1.5 m with occasional cobbles below 3.5 m. At Borehole 32, auger refusal was met within the fill at a depth of 1.0 m.

4.4 Fill At or Beyond Embankment Toe

Fill was encountered at surface in Boreholes 39, 40 and 41 varying in thickness from 0.6 to 1.0 m. The fill consists of 0.3 to 0.45 m of topsoil overlying sand fill with rockfill and/or organics extending to depths of 1.0 to 3.1 m in thickness.

The Standard Penetration Test (SPT) results generally indicate very loose to loose state of denseness (N values vary from 1 blows/0.3 m to 4 blows/0.3 m).

A significant organic content was identified within the sand fill at Boreholes 39, 40 and 41 (within close proximity of the propose culvert extension). The organic content was measured to be up to 19% with a natural moisture content as high as 160 %.

4.5 Organics

Only a trace of organics was found (in the split spoon sample from Boreholes 33 and 34) beneath the road fill, at 2.4 m and 3.8 m depth respectively.

Off the highway, a peat/organic layer is present at surface in Boreholes 35 to 38 and below surficial topsoil at Borehole 42. The organic layer varies from Elevation 170.5 to 170.8 m and varies in thickness from 0.8 m to 1.4 m. The organic layer consists of a mixture of root mat, inorganic soil and peat.

Primary and secondary consolidation properties of the organics have been estimated based on published correlations for peat.

Based on published experience with peat soils, a drained angle of internal friction of 30° has been estimated for design purposes. This applies to large deformations which eliminate the effects of fibers.

4.6 Sand and/or Silt

From beneath the fill and/or organics noted above, interbedded layers of sand and silt exist to auger

refusal varying between 3.8 and 10.2 m below existing grade. Gradation analyses conducted on representative samples retrieved from the field investigation at Boreholes 33 and 34 indicate 0% gravel content, 92% to 93% sand and 7 to 8% silt (Enclosure 18).

The Standard Penetration Test (SPT) results generally indicate a very loose to compact state of denseness (N values vary from 2 blows/0.3 m to 21 blows/0.3 m). At Boreholes 35 and 36 at depths of 3.0 and 4.5 m, N values of 72 and 36 blows for 0.30 m were achieved indicating a dense condition.

4.7 Clay

A discontinuous silty clay seam was identified at Borehole 44 starting at a depth of 0.75 m and extending to a depth of 1.5 m. Atterberg limits carried out on a sample of the clay from Borehole 44 at a depth of 0.75 m indicates the clay is of low plasticity (Enclosure 20). The natural moisture content of 29% approaches the liquid limit of 30%.

The Standard Penetration Test (SPT) results indicate at least a firm consistency (N value of 5 blows/0.3 m).

4.8 Groundwater

The groundwater levels taken on completion of drilling are noted on the Borehole Logs, Enclosures 1 to 15. The groundwater level in the off road boreholes noted during our field investigation varied between 0.1 m and 1.5 m below existing grade. The majority of the recordings are within 0.3 m from existing grade. For design purposes, the groundwater level should be taken at the natural ground surface outside the embankment.

Groundwater levels may fluctuate seasonally and in response to climatic conditions.

5.0 DISCUSSION

DST Consulting Engineers Inc. (DST) has been retained by JMC Transportation Group to conduct a foundation investigation for proposed west bound passing lane of Highway 17 in the Township of Head between Stations 30+750 and 31+100 and the culvert extension at Station 30+930, as defined by the Ministry as the Foundation Investigation and Design Report. It is understood that no significant change in grade is expected. The existing embankment width (shoulder to shoulder) is understood to be 12.1 m. With the addition of a new passing lane and the proposed modifications to the lane and shoulder widths, it is understood that the new embankment width from end of shoulder to end of shoulder will be 16.55 m. With a 1.0 m rounding proposed for the passing lane side of the embankment, the distance from existing centerline to end of rounding on the passing lane side will be 11.5 m ($3.75 + 3.75 + 3.0 + 1$). Along the proposed passing lane side, the existing height of the embankment varies from 0.8 to 4.0 m with side slope grades between 1.7 and 6.5 horizontal to 1 vertical.

The subsurface conditions below the existing embankment shoulder consists of granular fills overlying sands and silts. In general, the subsurface conditions beyond the existing toe of the embankment were found to consist of up to 1.4 m of peat/organics overlying sands and silts.

Engineering properties for embankment FILL has been estimated as follows:

- Unit Weight, $\gamma = 20 \text{ kN/m}^3$
- Drained Angle of Internal Friction, $\phi' \geq 30^\circ$
- Drained Cohesion Intercept, $c' = 0 \text{ kPa}$

Engineering properties for FILL material at embankment toe has been estimated as follows:

- Unit Weight, $\gamma = 20 \text{ kN/m}^3$

- Drained Angle of Internal Friction, $\phi' \geq 28^\circ$
- Drained Cohesion Intercept, $c' = 0$ kPa

Consolidation properties for this sand and organic fill has been based on published correlations for peat and have been estimated for the highest natural moisture content measured.

- Void Ratio (e) varies from 1 to 3.
- Compression Index (C_c) varies from 1.0 to 1.5.
- Secondary Compression Index (C_α) varies from 0.1 to 0.15.
- Unit Weight (γ) = varies from 11.0 to 12.5 kN/m³

For the normally consolidated organics outside of the existing embankment the following consolidation properties have been estimated based on highest natural moisture content measured of 797%:

- Void Ratio (e) varies from 12.4 to 14.9 .
- Compression Index (C_c) varies from 6.0 to 8.7.
- Secondary Compression Index (C_α) varies from 0.6 to 0.87.
- Unit Weight (γ) = varies from 10.3 to 10.7 kN/m³.

Engineering properties for sand or silt have been estimated as follows:

- Unit Weight, $\gamma = 20$ kN/m³
- Drained Angle of Internal Friction, $\phi' \geq 28^\circ$
- Drained Cohesion Intercept, $c' = 0$ kPa

Engineering properties for clay have been estimated as follows:

- Unit Weight, $\gamma = 18 \text{ kN/m}^3$
- Drained Angle of Internal Friction, $\phi' \geq 30^\circ$
- Drained Cohesion Intercept, $c' = 0 \text{ kPa}$
- Undrained Cohesion, $c_u \geq 25 \text{ kPa}$
- Void Ratio (e) = 0.75.
- Compression Index (C_c) = 0.2.

Based on the findings of the field and laboratory programs preliminary engineering assessment of various construction alternatives were carried out to identify the most feasible construction option. The results of this preliminary assessment have been discussed in Section 5.1, Preliminary Assessment of Construction Alternatives. Assessments for stability and settlement have been carried out for static conditions and are reported in subsequent sections of this report for the preferred option.

As indicated in the Draft Supplemental Pavement Design Report, G.W.P. 392-98-00, carried out for this project by Golder Associates Ltd., it is understood that there are no settlement areas or distortions within this area of the project which could be attributed to frost heave or to the presence of compressible layers beneath the pavement.

The culvert at Station 30+930 is understood to be a 3.65 x 1.7 x 22.7 concrete rigid frame culvert. The reported obverts are Lt. 171.57 m and Rt. 171.56 with the flow of Colton Creek flowing from left to right. The creek level at the time of this investigation was 170.63 m and the ditch invert on the downstream (left) end of the culvert was reported at 170.02 m.

The stratigraphy at the culvert location consists of granular fills with trace organics to a depth of 2.3

m at the existing shoulder and fills comprised of topsoil and sand with organics to a depth of 2.1 m underlain by sands and silts within the limits of the proposed culvert extension.

5.1 Preliminary Assessment of Foundation Alternatives

Based on the findings of the field investigation works and engineering assessment, the following table has been prepared summarizing and comparing various foundation alternatives. Through this exercise of comparing the various foundation alternatives, DST has selected a preferred option.

DST recommends subexcavation and replacement of the peat below the proposed embankment widening (Foundation Alternative 2 in the following table). Given relatively shallow, but highly variable peat thickness found below the proposed widening, peat removal is recommended to avoid potential differential and excessive embankment settlements. As this peat thickness is generally less than 1.3 m, costs associated with peat removal are considered to be relatively small given the much improved settlement performance. The subsequent sections of this report address this foundation alternative in detail.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
1) Fill placement without peat excavation	Lowest cost alternative.	Poor settlement performance with highly differential and short and long term settlements given highly variable peat thicknesses. Within the embankment widening, maximum settlements due to primary consolidation of the peat has been estimated at 1100 mm, most of which will occur over a period of 1 to 2 months (regrading will be required to account for settlement during this period). Additional settlement due to secondary consolidation of the peat over a 25 year period has been estimated to be in the order of 125 mm. Side slopes flatter than 1.25:1 for rockfill and 2:1 for earth fill would likely be required. Possible peat displacement during fill placement at localized areas; however this risk can be reduced through the use of a geotextile. Typically, staged construction may be required where embankment fills over peat exceed 1.2 m which could add to construction delays.	Lowest cost for construction. Some additional construction costs associated with geotextile and staged construction may be warranted to avoid peat displacement. Highest long term maintenance costs.	High risk of poor settlement performance (both short and long term). Instability and displaced peat will need to be addressed during construction. Zones where peat has been displaced can lead to more severe differential settlements. Risk of higher long term maintenance costs associated with settlements.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
2) Fill placement with peat sub-excavation	Short and long term settlement below widening associated with primary and secondary consolidation of the peat is mitigated. Settlements associated with native soils below peat are expected to be negligible. Conventional fore slopes of 1:25:1 for rockfill and 2:1 for earth fill will likely prove adequate.	Excavation of the peat will likely be required in the wet with subsequent fill placement. Fill placement below water will require the use of 19 mm stone or rockfill and a geotextile separator.	Costs associated with peat excavation should be considered, but are not expected to be excessive given the peat thickness varies from nil to 1.4 m below the proposed widening (as indicated on the test hole logs). It is expected that excavation can be carried out without utilizing specialized equipment such as a drag line. Long term maintenance costs will be reduced given improved settlement performance.	It will be important to ensure all peat is removed especially where excavation in the wet is required. Where zones of peat are left in-place localized areas of differential settlement can be expected. Should peat be found at depths deeper than expected special measures may be required for subexcavation. Potential for instability and/or settlement on existing embankment due to adjacent subexcavation and fill placement.
3) Lightweight fills without peat excavation	Can be used to mitigate/reduce potential settlement issues.	Unless costly rigid foam type insulation is considered for lightweight fill, settlement reduction will be negligible. Foam type insulation is not recommended below the water level. Some fill will be required above and below the insulation which will lead to primary and secondary consolidation of the peat. For up to 1.0 m of fill placement primary consolidation in the order of 260 mm can be expected with secondary consolidation settlements in the order of 150 mm over 25 years. Regrading will be required during the period of primary consolidation.	Construction utilizing rigid foam type insulation is likely cost prohibitive and not warranted. Some increased long term maintenance cost may be realized to address some differential settlement due to secondary consolidation of the peat.	Black ice formation is a known problem where rigid foam type insulations are used.

Foundation Alternative	Advantages	Disadvantages	Costs	Risks/Consequences
4) Preload with or without surcharge or wick drains	Preloading without wickdrains may be considered to reduce/mitigate primary consolidation of the peat. Wickdrains would not prove economical to accelerate preload duration. Light surcharge loads may be considered.	Preloading would require careful management as fills will need to be imported to restore the grade during preloading. The addition of fill will initiate additional primary consolidation which can extend preload times. Preload durations in excess of 1 to 2 months may be required to effectively eliminate primary consolidation. Secondary consolidations in the order of 125 mm over 25 years would still be expected. A surcharge may be considered to reduce the expected secondary consolidations, however, this would add to the risk of instability of the embankment and would likely require staged construction to place the preload and surcharge adding to construction delays. As such, only light surcharge loads are considered feasible.	Additional cost associated with additional regrading required after preloading and for construction delays. Long term maintenance cost will be higher than that of peat removal given some long term settlement can be expected due to secondary consolidation (unless a surcharge is considered). Displacement of the peat may be incurred during construction.	Preload may not prove adequate in eliminating all long term consolidation of the peat which can be highly differential leading to increased maintenance costs.
5) Retained Soil System	Can be useful to permit construction with steeper than conventional slopes. However, given embankment toes are expected to be within existing ROW, an RSS is not warranted.	High costs, not warranted. Would likely require peat removal to provide necessary foundation capacity and improve settlement performance.	High cost, not warranted.	A retained soil system does not improve settlement performance, not warranted.

5.2 Stability Analysis For Proposed Passing Lane

The grade of the proposed new passing lane and associated shoulder will approximately match the height of the existing roadway (approximately 0.8 to 4.0 m height) and be 6.75 m in width. The existing side slope is constructed at a slope of 1.7 to 6.5 H:1V utilizing granular fill. At this time it is not known if the proposed new widening will utilize rockfill (if available), or granular fill. As such, recommendations for both construction materials have been provided. Where rockfill will be utilized, a side slope of 1.25H:1V has been proposed. Where granular fill (such as Granular "B", Type 1) will be used, a side slope of 2H:1V has been proposed. Peat subexcavation has been proposed as per OPSD 203.030, or OPSD 203.020 (Appendix 'B'). The peat subexcavation is to be extended as per Note 1 of these standards.

The above noted configurations provide a minimum design target factor of safety of at least 1.3 for end of construction and final long term drained conditions.

Under OPSD 203.020, the existing side slope is specified to be temporarily cut at a slope of 1 horizontal to 1 vertical starting at the existing shoulder. This is done to facilitate the removal of more peat possibly located below the existing side slope. It should be noted that sloughing of the 1 horizontal to 1 vertical slope may be experienced during construction. The degree of sloughing will be largely dependant on the duration in which it is left to stand. Backfilling of the slope to design levels should commence immediately after excavation. We recommend rapid excavation and backfilling methods be utilized. The temporary slope should be visually inspected as construction progresses. Should excessive sloughing become problematic, the temporary cut slope angle may be revised to 1.5 horizontal to 1 vertical. However, additional settlements may be incurred below the proposed new shoulder and rounding.

5.3 Settlement of Proposed Passing Lane

Maximum settlements induced by the proposed passing lane fills due to primary and secondary consolidation of any peat remaining below the existing embankment and side slope have been estimated based on two scenarios as follows:

1. During peat subexcavation, the existing side slope is maintained, as per OPSD 203.030.
2. During peat subexcavation, the existing side slope is cut at 1H:1V, as per OPSD 203.020.

The following table indicates the settlements calculated for construction as per OPSD 203.030.

Location	Primary Consolidation of Peat (mm)	Secondary Consolidation of Peat Over 25 yrs. (mm)	Total Settlement After 25 years (mm)
C/L Existing Roadway	N/A	N/A	N/A
Existing Shoulder	<5	50	55
Midpoint Passing Lane	<5	50	55
Passing Lane Shoulder	125	125	250

The following table indicates the settlements calculated for construction as per OPSD 203.020.

Location	Primary Consolidation of Peat (mm)	Secondary Consolidation of Peat Over 25 yrs. (mm)	Total Settlement After 25 years (mm)
C/L Existing Roadway	N/A	N/A	N/A
Existing Shoulder	<5	50	55
Midpoint Passing Lane	<5	50	55
Passing Lane Shoulder	N/A	N/A	N/A

Estimated settlements due to consolidation of organic soils for the above scenarios have been presented in the following tables. The settlements have been calculated for a passing lane embankment constructed to the same grade as the existing highway shoulder over the thickest deposit of peat found below the proposed embankment widening. Settlements have been calculated at the existing centreline, existing shoulder, approximate midpoint of passing lane, and

the approximate shoulder of the new passing lane. It is expected that most of the primary consolidation of the peat will occur over a period of 1 to 2 months. For a 2 month construction period, we expected that at least 90% of the primary consolidation will have occurred (but will depend on regrading and leveling activities during this period). Secondary consolidation has been calculated over a 25 year period.

The settlements along the longitudinal profile of the passing lane will vary and be roughly proportional to the thickness of the peat deposit below the embankment.

Settlements attributed to the native sands and silts below the proposed widening have estimated to be less than 50 mm and are expected to occur during construction.

6.0 RECOMMENDATIONS

6.1 Passing Lane

A new westbound passing lane is proposed from Stations 30+750 and 31+100 in the Township of Head. The existing highway embankment is approximately 1 to 4 m in height and is to be widened to facilitate the construction of the new passing lane and shoulder.

The additional load of the proposed passing lane does not adversely affect the stability of the existing highway embankment, nor the passing lane itself. The construction of the passing lane can therefore proceed without special considerations for stabilizing the embankment. It is recommended the passing lane be constructed in accordance with OPSD 203.020, Embankments Over Swamp, Existing Slope Excavated to 1:1, to avoid potentially excessive and differential settlement along the proposed shoulder (due to consolidation of the peat underling the existing side slope).

It is expected that at least 90% of the primary consolidation settlements within organic soils, as provided in Section 5.3, will occur during construction. The remaining long term settlements will begin during fill placement for the passing lane and will continue at a decreasing rate. As such, some minor long term maintenance of these areas may be required as the new fills continue to settle. This may involve occasional grading/filling of shoulders and patching of asphalt.

The widening of the roadway should be in accordance with OPSD – 203.020 Embankment Over Swamps, Existing Slope Excavated to 1:1. Benching of earth slopes should be carried out in accordance with OPSD 208.010, Benching of Earth Slopes (Appendix 'B'). The existing organics and deleterious material beneath the proposed widening should be excavated to the native sand which is expected to be up to 1.4 m deep as indicated as discrete borehole locations. The width of swamp excavation should include the additional excavation as indicated in Note 1 of

OPSD 203.020.

The native sand subgrade will likely be wet and easily disturbed through site traffic of any type. Water ingress into the excavation will need to be controlled and granular fill should be placed and compacted immediately following excavation. Where dewatering can not be accomplished, excavation of the peat and fill placement in the wet should be considered. The use of self compacting clear stone (such as 19mm stone) or rockfill should be used below the water level.

Where the embankment material will consist of rockfill, a side slope of 1.25H:1V can be constructed. Voids on the top of the rockfill embankment should be chinked with rock fragments and spalls to form the subgrade as per OPSS 206.07.08, Rock Embankments.

Where the embankment material consists of Granular 'B', Type 1 fill in accordance with OPSS 1010, a side slope of 2:1 should be constructed. The embankment fill should be placed in lifts to the required subgrade level and compacted to a minimum 95% of standard Proctor maximum dry density. In order to achieve the specified compaction to the slope face it may be necessary to overbuild the slope and then cut it back to design grades.

Where rockfill and/or clear stone fill is used to replace the excavated peat, a non-woven geotextile (OPSS 1860.07.05.02 Class II) should be placed between the native subgrade and fill to prevent the migration of fines into the void spaces of the fill.

The stability of the proposed culvert extension and adjacent embankment require that the existing embankment overly non-liquefiable fills (clear stone) to an elevation of at least 166.0 m. Details are provided in Section 5.4.

6.2 Culvert Foundation (Station 30+930)

The culvert at Station 30+930 is understood to be a 3.65 x 1.7 x 22.7 concrete rigid frame culvert. As reported by K Smart Associates, the obverts are Lt. 171.57 m and Rt. 171.56 with the flow of Colton Creek flowing from left to right. It is understood that the top of footing elevation is 1.84 m below the obvert elevation. Assuming a footing thickness in the order of 0.3 m, the elevation of the existing base of existing is approximately 169.5 m. The creek level at the time of this investigation was 170.63 m and the ditch invert on the downstream (left) end of the culvert was reported at 170.02 m. Through consultation with K. Smart Associates, it is understood that the existing footing widths are likely in the order of 0.5 to 1.0 m.

6.2.1 Shallow Foundations – Strip Footings

Given the presence of loose sand and organic fill within the limits of the proposed culvert extension, it is recommended that all existing fill and/or organics be stripped from below the footprint of the proposed extension to avoid excessive and differential settlements. Based on the finding of Borehole 40, the deleterious materials are expected to extend to an elevation of about 168.5 m (approximately 0.9 m below the existing footing elevation). The excavated fills and organics are to be replaced with 19 mm clear stone to facilitate construction in the wet without compaction. A non-woven geotextile (OPSS 1860.07.05.02 Class II) must be placed between the native subgrade and clear stone fill to prevent the migration of fines into the void spaces of the fill which can lead to settlement.

Strip footings between 0.5 and 1.0 m in width may be designed based on geotechnical resistances in terms of limit states as provided in the following tables. For the provided geotechnical resistances, the footings must be supplied with a minimum depth of cover of 0.3 m and are to be

underlain with a minimum thickness of 19 mm clear stone as specified. The base of the clear stone must extend beyond the edge of footing a distance equal to the thickness of fill below the footing. It should be noted that a thicker pad thickness of 19 mm clear stone is required to address seismic concerns as discussed in Section 5.4.2 of this report.

Strip Footings Underlain with 0.5 m (minimum) of 19 mm Clear Stone

Factored Bearing Resistance at ULS	160 kPa
Bearing Resistance at SLS	100 kPa (25 mm settlement)
Bearing Resistance at SLS	205 kPa (50 mm settlement)

Strip Footings Underlain with 1.0 m (minimum) of 19 mm Clear Stone

Factored Bearing Resistance at ULS	200 kPa
Bearing Resistance at SLS	135 kPa (25 mm settlement)
Bearing Resistance at SLS	275 kPa (50 mm settlement)

The design values for strip footings are for vertical concentric loads in compression. Should eccentric loads be realized, the footing size may have to be increased to provide the minimum effective footing width as specified and/or the footing location adjusted with respect to the structural loads. Reduction factors for inclined loads must also be considered.

Slope stability analysis for the culvert extension was carried out for a potential slip surface extending through the open base of the culvert. The results of this analysis indicate a satisfactory safety factor of stability.

The horizontal resistance of the footing may be estimated in accordance with section C6-8.4.3 of the Commentary for the Ontario Highway Bridge Code. For the above configuration, horizontal resistance will be provided along footing-soil interface. Passive resistance of the sloped clear stone should be neglected. The resistance along the footing-soil interfaced may be calculated assuming $\tan \delta = 0.55$.

6.2.2 Seismic Foundation Assessment

The maximum design level earthquake for this site has been selected as one which results in ground accelerations for a 10% probability of exceedance in 50 years. As per the Canadian Highway Design Bridge Code (2000), Deep River is located in an acceleration related seismic zone 4. The range of peak horizontal ground acceleration varies from 0.16 to 0.23g for the above noted probability. A peak horizontal ground acceleration of 0.23 has been used for the following seismic assessment with a design earthquake magnitude of 7.5.

A seismic liquefaction assessment has been carried out for the proposed foundation of the culvert and associated extension. This assessment has been based on the findings at Borehole 33 (located on the left shoulder adjacent the existing culvert), Borehole 40 (located near the left limits of the proposed extension), and Boreholes 39 and 41. The results of this liquefaction assessment indicate the native soils below the culvert will liquefy (with factors of safety as low as 0.2) extending down to elevations of 164.2 to 166.1 m (up to 5.3 m below the base of footing). As such, further analyses considered flow slides, lateral displacements, ground damage and bearing capacity failures during seismic induced liquefaction.

Due to the potential for this zone of soil to liquefy and subsequently loss in strength during a design seismic event, a weakening slope stability analyses was carried out to consider the potential for a flow slide for the embankment adjacent to the culvert. The results of this analysis indicate a significant risk for flow slides. To limit this risk, the upper zone of the potentially liquefiable soils may be replaced with non-liquefiable soil such as 19 mm clear stone wrapped with a non-woven geotextile. The non-liquefiable zone, consisting of 19 mm clear stone, must extend to elevation 166.0 m (approximately 3.5 m below underside of footing) and extend to a distance of 4.0 m beyond the embankment toes. Based on the findings at the existing shoulder (Borehole 33), it is most likely

that liquefiable soils exist below the existing culvert. As such, the existing culvert will need to be temporarily removed and/or replaced to provide the required zone of non-liquefiable clear stone. The alternate use of dynamic compaction, and/or vibro-compaction to improve the liquefaction resistance of the underlying soils would likely lead to excessive and highly differential settlements of the existing culvert and are not recommended.

Given the extensive zone of potentially liquefiable soils, an assessment of lateral spreading (as opposed to flow slides) was carried out. The results of this assessment indicate horizontal ground displacements resulting from liquefaction induced lateral spreading is expected to be negligible (less than 5 mm).

To protect the culvert from liquefaction induced ground damage due to the potential occurrence of sand boils or surface fissures, the footing must not be allowed to sit directly on potentially liquefiable soil. The minimum thickness of non-liquefiable soil below the foundation element has been estimated to be 2.5 m. This is less than the thickness required to limit the risk of flow slides for the adjacent embankment and as such, damage due to sand boils or surface fissures is not expected.

Given a zone of liquefiable soil exists below the footing elements, a bearing capacity analyses for liquefied soil has been carried out. A punching shear analyses was carried out for with a minimum thickness of 3.5 m of 19 mm clear stone (as required for flow slides) modeled between the base of footing and top of the underlying liquefied soils. Under these conditions, an allowable footing load of 80 kN/m may be permitted. This load should consider dead, live and seismic loads acting on the footing as well as the weight of footing itself. Foundation settlements induced due to the liquefaction of the remaining liquefiable soils below the foundation have been estimated at 70 mm.

It is likely that conditions under the proposed culvert extension extend under the existing culvert as well. This means that the existing culvert and adjacent embankment do not have sufficient stability during the design seismic event. If this is considered an acceptable risk, then basing the design of the culvert extension on static conditions may be considered. However, under this scenario, the stability of the existing culvert and extension remain a potential risk during a design seismic event until such time as remedial and/or replacement designs have been developed.

6.2.4 Lateral Earth Pressure

Earth pressures should be computed as per Section 6.9.2 of the Canadian Highway Bridge Design Code (CHBDC). Granular "A" or "B", Type 1 backfill should be in accordance with Ontario Provincial Specifications (OPSS 1010). The following parameters are recommended when calculating active or at rest earth pressures.

	Granular "A"	Granular "B", Type 1	Rockfill
Angle of Internal Friction	$\phi' = 35^\circ$	$\phi' = 30^\circ$	$\phi' = 40^\circ$
Unit Weight (kN/m ³)	$\gamma = 23$	$\gamma = 21$	$\gamma = 18$

6.2.5 Frost Protection

In accordance with the Pavement Design and Rehabilitation Manual, the frost penetration depth for this project is 2.0 m.

Where footings have inadequate soil cover, the actual degree of potential stresses and heave cannot be accurately predicted, although it is noted that the existing culvert has performed adequately.

Rockfill and rock protection shall count for half of their thickness in determining the depth of cover

provided.

It should be noted that the foundation recommendations for the culvert require a minimum thickness of non-frost susceptible 19 mm clear stone below the footing. This material will not be subject to the formation of ice lenses and excessive heave. However, due to the volumetric expansion of water within the void space, relatively minor heave movements may be expected should this fill be allowed to freeze. The native soils founded below the clear stone pad are considered to be frost susceptible.

6.2.6 Scour Protection

The footings should be provided with sufficient scour protection to the minimum specified depth of cover and frost protection is preserved and to ensure footings are not undermined. Scour protection should be in accordance with Section 1.10 of the CHBDC.

6.3 Construction

The construction methodology must be in accordance with all relevant Ministry guidelines. The contractor's methods and equipment must be suitable for the site conditions and materials used.

Equipment and worker traffic over the subgrade soil should be avoided and the first lift of fill should be placed and compacted immediately after excavation to provide a working surface and protection for the subgrade. Given the current groundwater level is at or near the surface of the peat, advanced dewatering methods may be required during excavation of the peat to permit excavations in the dry. It may prove more economical to carry out peat subexcavation and fill placement in the wet. Where construction in the wet is considered, tight quality control will be required to ensure adequate subexcavation has been achieved.

7.0 LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix "A", and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:

Reviewed by:

R.F. Crowley, P. Eng.
Sr. Project Engineer

Mike Fabius, P. Eng.
Principal

RFC:dm

APPENDIX 'A'
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

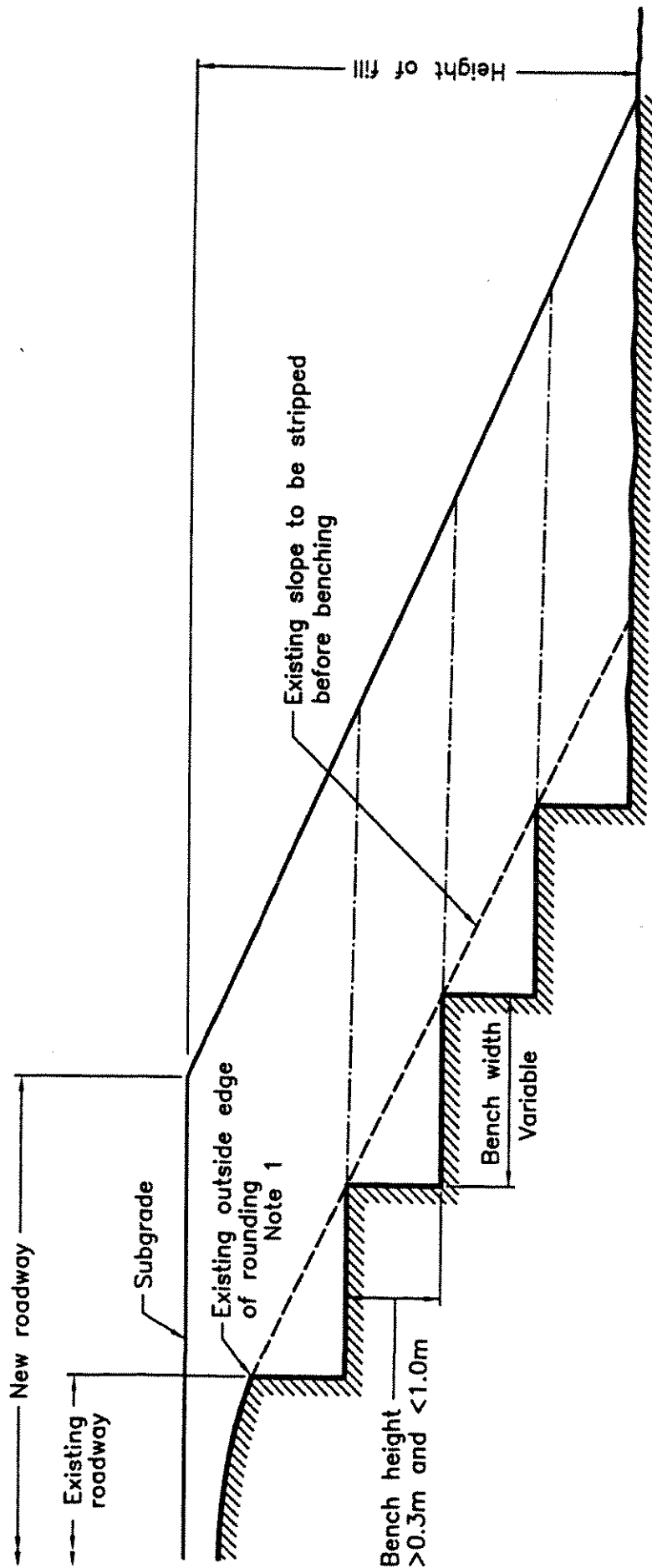
Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

A P P E N D I X 'B'

OPSD DRAWING



NOTES:

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

A Benching is not required on existing slopes flatter than 3H:1V.

B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

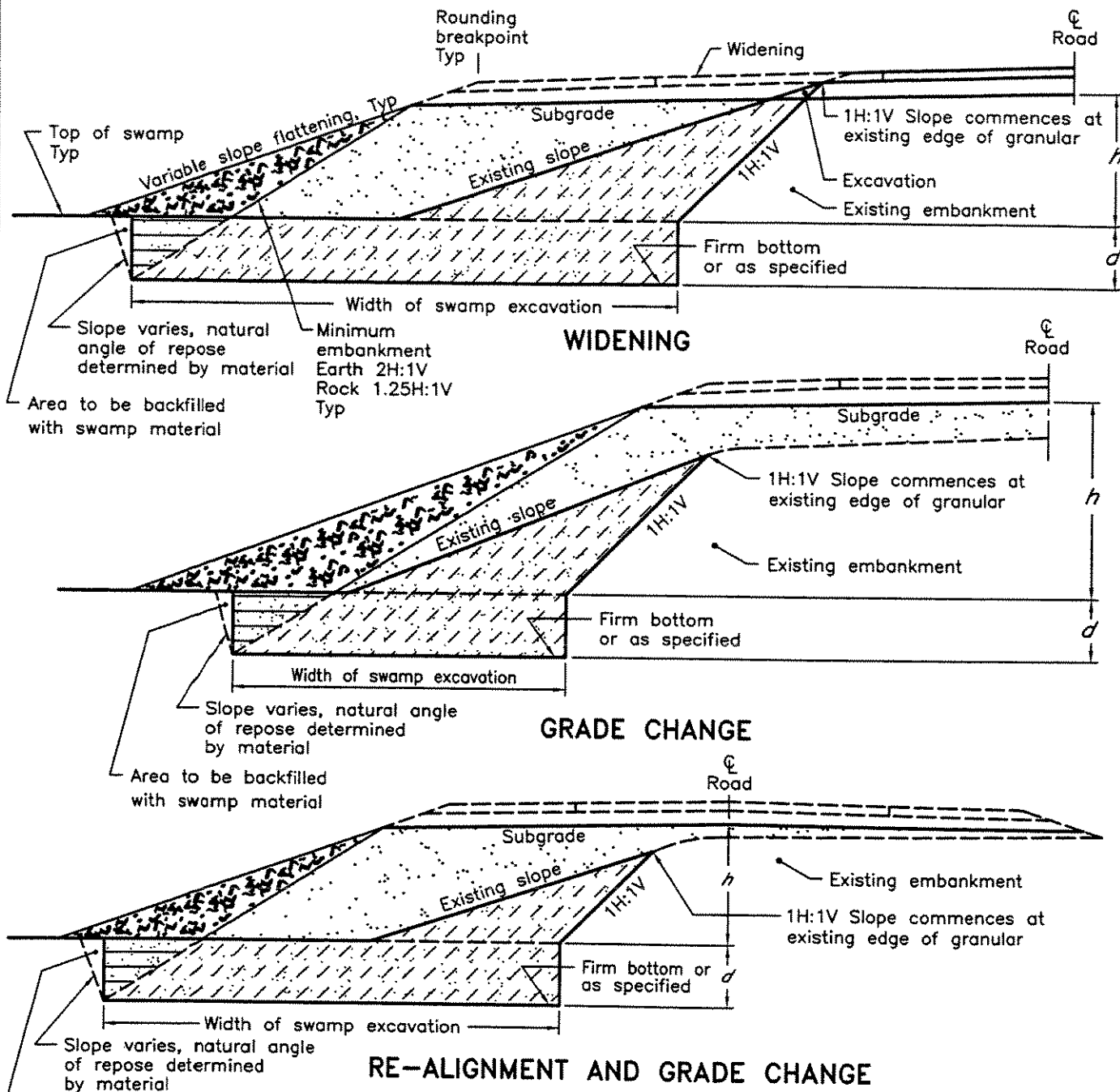
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2003 Rev 1



BENCHING OF EARTH SLOPES

OPSD - 208.010



NOTES:

- For given limits of height $h \leq 4.5\text{m}$ and depth $d \leq 6.0\text{m}$, both requirements shall be met in order to apply.
- Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.
- Widening of existing earth embankments shall be benched according to OPSD-208.010.
- All dimensions are in millimetres unless otherwise shown.

LEGEND:

- h - Height of fill
- d - Depth of sub-excitation
- Embankment materials as specified
- Excavated swamp material
- Excavate and backfill
- Excavate and backfill with swamp material

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2004

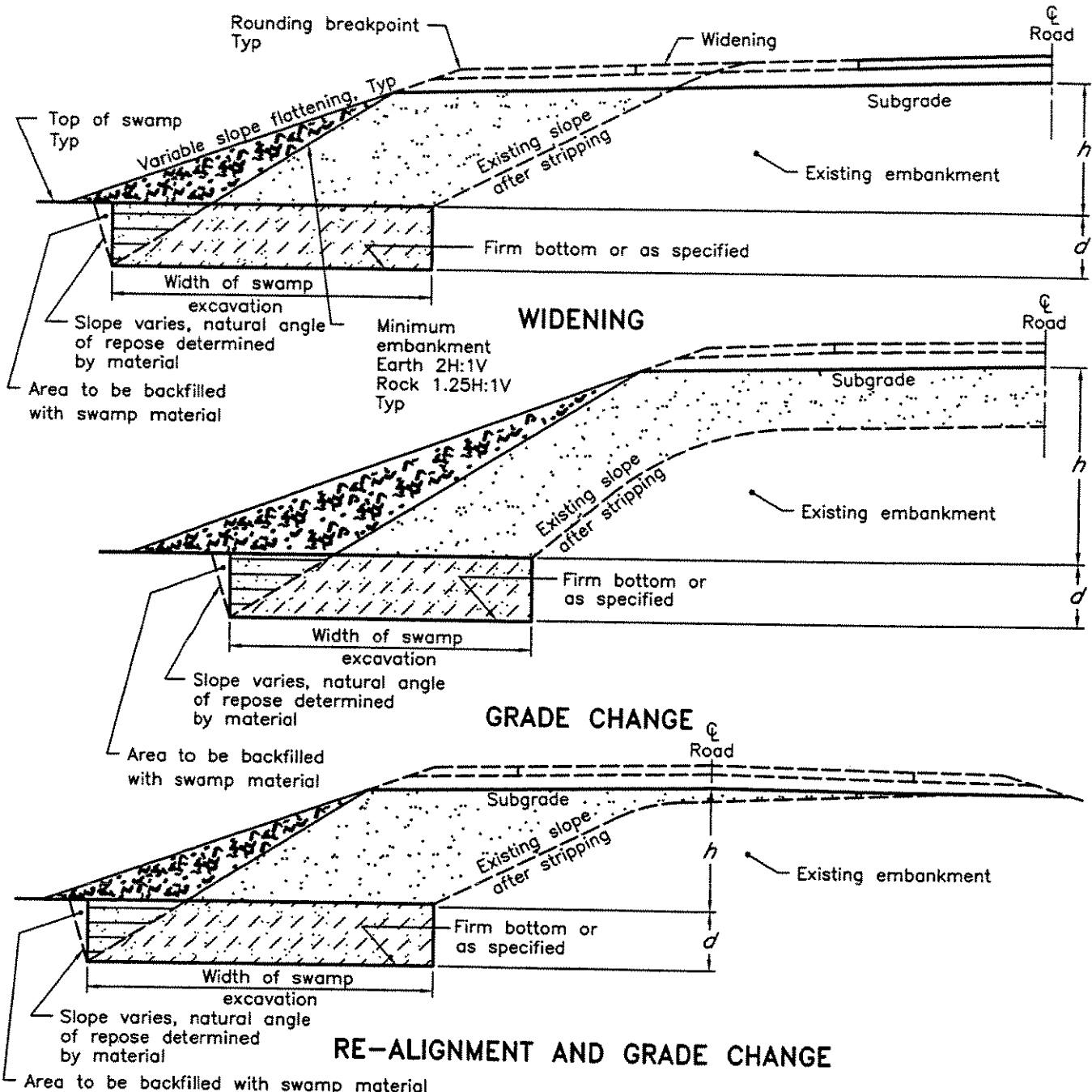
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EMBANKMENTS OVER SWAMP

EXISTING SLOPE EXCAVATED TO 1H:1V

OPSD - 203.020





NOTES:

- For given limits of height $h \leq 4.5\text{m}$ and depth $d \leq 6.0\text{m}$, both requirements shall be met in order to apply.
- Topsoil shall be stripped from existing slopes.
- Height of fill is the vertical difference between top of subgrade and top of swamp elevation measured at new road centreline.
- Widening of existing earth embankments shall be benched according to OPSD-208.010.
- All dimensions are in millimetres unless otherwise shown.

LEGEND:

- h - Height of fill
 d - Depth of sub-excavation
- Embankment materials as specified
 - Excavated swamp material
 - Excavate and backfill
 - Excavate and backfill with swamp material

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2004 Rev 1

EMBANKMENTS OVER SWAMP

EXISTING SLOPES MAINTAINED



OPSD - 203.030

DRAWINGS

ENCLOSURES

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	KN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	KN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	KN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	KN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	KN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	KN/m^2	SEEPAGE FORCE
γ'	KN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						